CHAPTER 4

STRUCTURE STABILITY

4-1. Scope. This chapter presents information for stability analysis of retaining walls and inland and coastal flood walls. The methods of analysis to determine overturning and sliding stability and to assess bearing capacity are discussed. The forces as determined in Chapter 3 are used to assess overturning stability and bearing capacity. In certain cases as described in this chapter, the same forces computed for overturning may be used to check sliding stability. In other cases, sliding stability should be computed by the multiple wedge iterative method or by an adjustment of the driving and resisting wedge forces based on the factor of safety required, both of which are discussed in this chapter. Loading conditions for the various types of walls and the acceptable criteria for each loading condition are given for each of the stability analyses.

Section I. Loading Conditions

4-2. Representative Loading Conditions. The following loading conditions are generally representative of conditions affecting retaining walls and inland and coastal flood walls. The loading cases for a specific wall should be chosen, as applicable, from the lists below. Loading conditions which are not listed below should be analyzed, where applicable. Note that some walls may require consideration of loadings from both lists, as discussed in paragraph 2-9.

4-3. Retaining Walls.

a. Case R1, Usual Loading. The backfill is in place to the final elevation; surcharge loading, if present, is applied (stability should be checked with and without the surcharge); the backfill is dry, moist, or partially saturated as the case may be; any existing lateral and uplift pressures due to water are applied. This case also includes the usual loads possible during construction which are not considered short-duration loads.

b. Case R2, Unusual Loading. This case is the same as Case R1 except the water table level in the backfill rises, for a short duration, or another type of loading of short duration is applied; e.g., high wind loads (paragraph 3-25), equipment surcharges during construction, etc.

c. Case R3, Earthquake Loading. This is the same as Case R1 with the addition of earthquake-induced lateral and vertical loads, if applicable; the uplift is the same as for Case R1.

4-4. Inland Flood Walls.

a. Case I1, Design Flood Loading. The backfill is in place to the final elevation; the water level is at the design flood level (top of wall less freeboard) on the unprotected side; uplift is acting.

b. Case I2, Water to Top of Wall. This is the same as Case I1 except the water level is at the top of the unprotected side of the wall.

c. Case I3, Earthquake Loading. The backfill is in place to the final elevation; the water is at the usual level during the non-flood stage; uplift, if applicable, is acting; earthquake-induced lateral and vertical loads, if applicable, are acting. (Note: This case is necessary only if the wall has a significant loading during the non-flood stage.)

d. Case I4, Construction Short-Duration Loading. The flood wall is in place with the loads added which are possible during the construction period, but are of short duration such as from strong winds (paragraph 3-25) and construction equipment surcharges.

4-5. Coastal Flood Walls.

a. Case C1, Surge Stillwater Loading. The backfill is in place to the final elevation; the water is at the surge stillwater level on the unprotected side; wave forces are excluded; uplift is acting.

b. Case C2a, Nonbreaking Wave Loading. This is the same as Case C1 with a nonbreaking wave loading added, if applicable; uplift is the same as for Case C1.

c. Case C2b, Breaking Wave Loading. This is the same as Case C1 with a breaking wave loading added, if applicable; uplift is the same as for Case C1.

d. Case C2c, Broken Wave Loading. This is the same as Case C1 with a broken wave loading added, if applicable; uplift is the same as for Case C1.

e. Case C3, Earthquake Loading. The backfill is in place to the final elevation; water is at the usual (non-storm) level; uplift, if applicable, is acting; earthquake-induced lateral and vertical loads, if applicable, are acting. (Note: If the wall has no significant load during the usual (non-storm) stage, no earthquake case is necessary.)

f. Case C4, Construction Short-Duration Loading. The flood wall is in place with the loads added which are possible during the construction period but are of short duration, such as from strong winds and construction equipment surcharges.

g. Case C5, Wind Loading. The backfill is in place to the final elevation; water is at the usual (non-storm) level on the unprotected side; a wind load of 50 lb/sq ft on the protected side of the wall is applied (paragraph 3-25).

Section II. Stability Considerations

4-6. General Requirements. Figure 4-1 illustrates the potential failure modes for which stability must be analyzed. The basic requirements for the stability of a retaining or flood wall for all loading conditions are discussed below.

a. The wall should be safe against sliding at its base, through any soil layer or rock seam below the base.

b. The wall should be safe against overturning at its base, and, in the case of gravity walls, at any horizontal plane within the wall.

c. The wall should be safe against bearing failure and excessive differential settlement in the foundation.

4-7. Stability Criteria. The stability criteria for retaining walls and inland and coastal flood walls are listed, by loading case, in Tables 4-1 through 4-3.

Section III. Overturning Stability

4-8. Resultant Location.

a. General Computations. To assess the overturning stability of a wall, such as the one with a horizontal base shown in Figure 4-2 (see examples 1, 2, 3, 5, 6, and 7 of Appendix N), all operative forces must be applied to a free body of the structural wedge wall/soil system. Methods to calculate the lateral and uplift forces are discussed in Chapter 3. The moments of these forces are summed about point O as shown in Figure 4-2 and the distance x_R is calculated as:

$$x_R = \frac{\text{summation of moments about Point O}}{\Sigma V} \qquad [4\text{-}1]$$

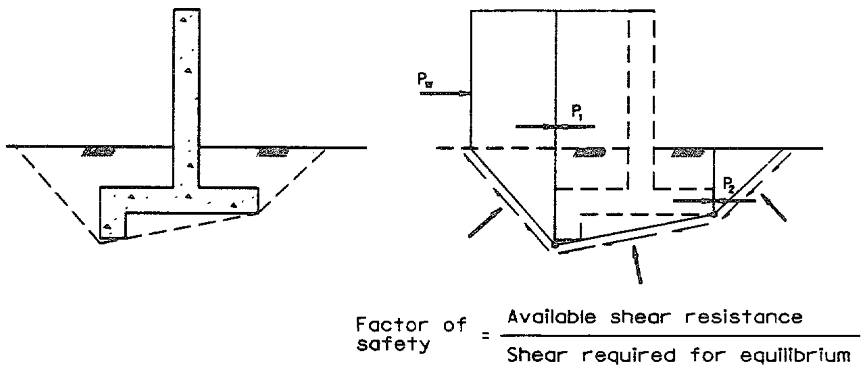
where

ΣV = resultant base force required for vertical equilibrium

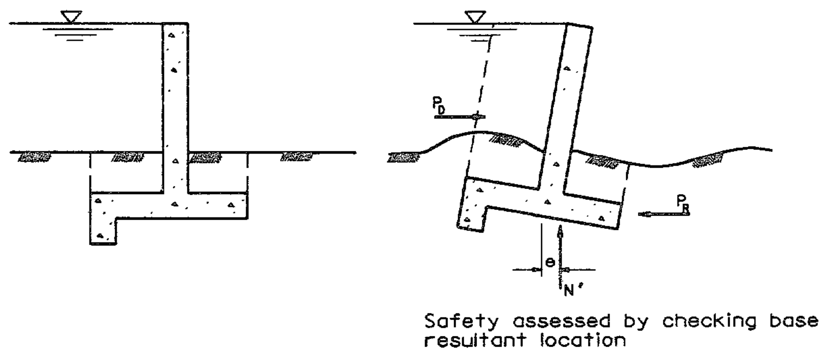
A ratio defined as the resultant ratio is computed as follows:

$$\text{Resultant ratio} = \frac{x_R}{\text{horizontal width of base}} \qquad [4\text{-}2]$$

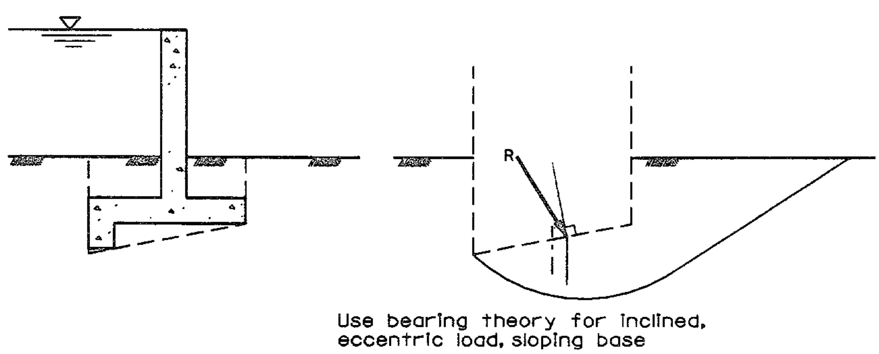
Equations 4-1 and 4-2 are valid for a wall with a horizontal base with or without a key and for a wall with a sloped base and a key. If a wall has only a sloped base (no key), as shown in Figure 4-3 (see example 4 of Appendix N), x_R is calculated as:



a. Sliding



b. Overturning



c. Bearing

Figure 4-1. Stability considerations for retaining and flood walls

Table 4-1

Retaining Wall Stability Criteria

Case No.	Loading Condition	Sliding Factor of Safety, FS	Shear Strength Test Required: Soil Foundation	Shear Strength Test Required: Rock Foundation[3]	Overturning Criteria Minimum Base Area in Compression: Soil Foundation	Overturning Criteria Minimum Base Area in Compression: Rock Foundation	Minimum Bearing Capacity Safety Factor
R1	Usual	1.5	(Q &/or S)[2,1]	Direct shear	100%[4]	75%[4]	3.0
R2	Unusual	1.33	(Q &/or S)[2,1]	Direct shear	75%[4]	50%[4]	2.0
R3	Earthquake	1.1	(Q)	Direct shear	Resultant within base	Resultant within base	>1.0

Notes

1. For soil foundations which are not free draining (permeability $< 10 \times 10^{-4}$ cm/sec), analyze for both Q and S strengths and design for the worst condition. For free-draining soil foundations (permeability $> 10 \times 10^{-4}$ cm/sec), analyze for S strengths only.

2. For construction loadings in Cases R1 and R2, use Q strengths when excess pore water pressure in the soil foundation is anticipated and S strengths when it is not anticipated.

3. The sliding analysis of a wall on rock should be based on the frictional resistance (tan ϕ) of concrete on rock or rock on rock. The values should be obtained from direct shear tests of pre-cut samples of concrete on rock and rock on rock, or direct shear tests of natural rock joints or bedding planes.

4. Less base area in compression than the minimum shown may be acceptable provided adequate safety against unacceptable differential settlement and bearing failure is obtained.

Table 4-2

Inland Flood Wall Stability Criteria

Case No.	Loading Condition	Sliding Factor of Safety, FS	Shear Strength Test Required: Soil Foundation	Shear Strength Test Required: Rock Foundation[3]	Overturning Criteria Minimum Base Area in Compression: Soil Foundation	Overturning Criteria Minimum Base Area in Compression: Rock Foundation	Minimum Bearing Capacity Safety Factor
I1	Design flood	1.5	(Q &/or S)[1]	Direct shear	100%[4]	75%[4]	3.0
I2	Water to top of wall	1.33	(Q &/or S)[1]	Direct shear	75%[4]	50%[4]	2.0
I3	Earthquake	1.1	(Q)	Direct shear	Resultant within base	Resultant within base	>1.0
I4	Construction	1.33	(Q &/or S)[2]	Direct shear	75%[4]	50%[4]	2.0

Notes

1. For soil foundations which are not free draining (permeability $< 10 \times 10^{-4}$ cm/sec), analyze for both Q and S strengths and design for the worst condition. For free-draining soil foundations (permeability $> 10 \times 10^{-4}$ cm/sec), analyze for S strengths only.

2. For construction loading cases, use Q strengths when excess pore water pressure in the soil foundation is anticipated and S strengths when it is not anticipated.

3. The sliding analysis of a wall on rock should be based on the frictional resistance ($\tan \phi$) of concrete on rock or rock on rock. The values should be obtained from direct shear tests of pre-cut samples of concrete on rock and rock on rock, or direct shear tests of natural rock joints or bedding planes.

4. Less base area in compression than the minimum shown may be acceptable provided adequate safety against unacceptable differential settlement and bearing failure is obtained.

Table 4-3

Coastal Flood Wall Stability Criteria

Case No.	Loading Condition	Sliding Factor of Safety, FS	Shear Strength Test Required: Soil Foundation	Shear Strength Test Required: Rock Foundation[3]	Overturning Criteria Minimum Base Area in Compression: Soil Foundation	Overturning Criteria Minimum Base Area in Compression: Rock Foundation	Minimum Bearing Capacity Safety Factor
C1	Surge stillwater	1.5	(Q &/or S)[1]	Direct shear	100%[5]	75%[5]	3.0
C2	Wave						
C2a	Nonbreaking	1.33	(Q &/or S)[1]	Direct shear	75%[5]	50%[5]	2.0
C2b	Breaking	1.25	(Q)[4]	Direct shear	60%[5]	40%[5]	1.5
C2c	Broken	1.33	(Q)[4]	Direct shear	75%[5]	50%[5]	2.0
C3	Earthquake	1.1	(Q)	Direct shear	Resultant within base	Resultant within base	>1.0
C4	Construction	1.33	(Q &/or S)[2]	Direct shear	75%[5]	50%[5]	2.0
C5	Wind	1.33	(Q &/or S)[2]	Direct shear	75%[5]	50%[5]	2.0

Notes

1. For soil foundations which are not free draining (permeability $< 10 \times 10^{-4}$ cm/sec), analyze for both Q and S strengths and design for the worst condition. For free-draining soil foundations (permeability $> 10 \times 10^{-4}$ cm/sec), analyze for S strengths only.
2. For construction loading cases, use Q strengths when excess pore water pressure in the soil foundation is anticipated and S strengths when it is not anticipated.
3. The sliding analysis of a wall on rock should be based on the frictional resistance ($\tan \phi$) of concrete on rock or rock on rock. The values should be obtained from direct shear tests of pre-cut samples of concrete on rock and rock on rock, or direct shear tests of natural rock joints or bedding planes.
4. For soil foundations which are not free draining (permeability $< 10 \times 10^{-4}$ cm/sec), analyze for Q test. For free draining soil foundations (permeability $\geq 10 \times 10^{-4}$ cm/sec), analyze for S strengths.
5. Less base area in compression than the minimum shown may be acceptable provided adequate safety against unacceptable differential settlement and bearing failure is obtained.

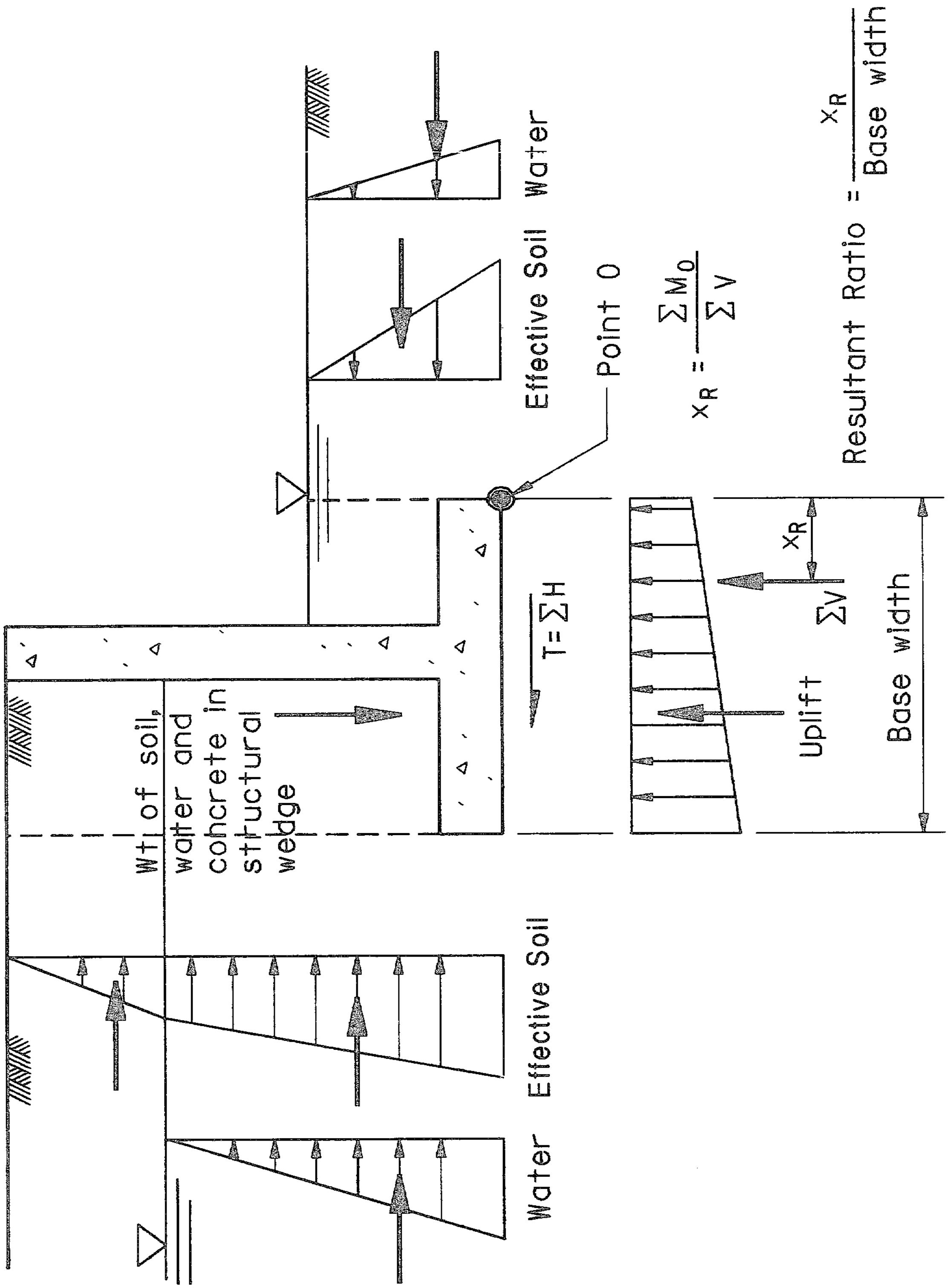


Figure 4-2. Forces for overturning analysis for wall with horizontal base

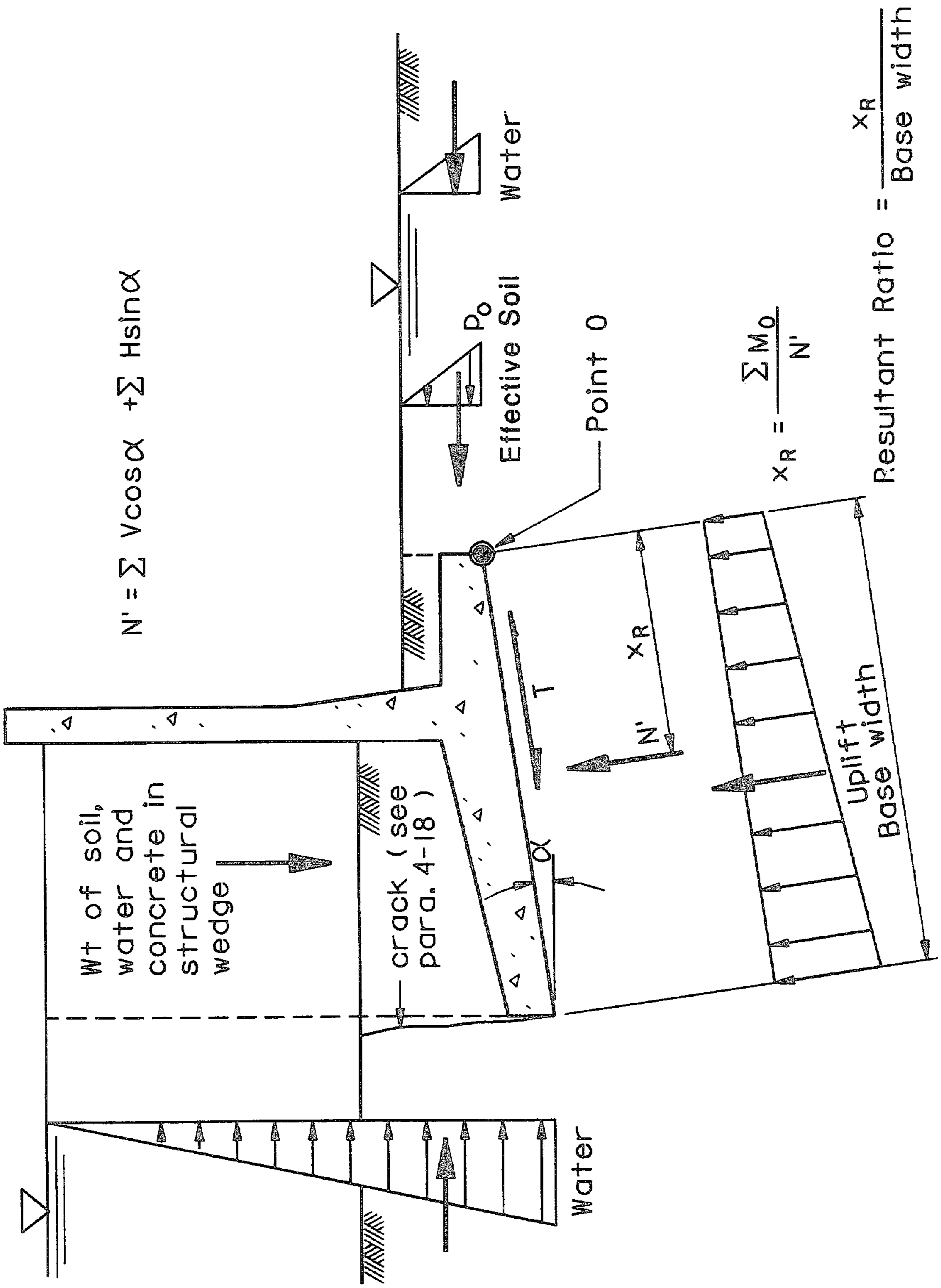


Figure 4-3. Forces for overturning analysis for wall with a sloping base

$$x_R = \frac{\text{summation of moments about Point O}}{\text{effective normal base force, N'}} \qquad [4\text{-}3]$$

The resultant ratio is defined as:

$$\text{Resultant ratio} = \frac{x_R}{\text{sloped base width}} \qquad [4\text{-}4]$$

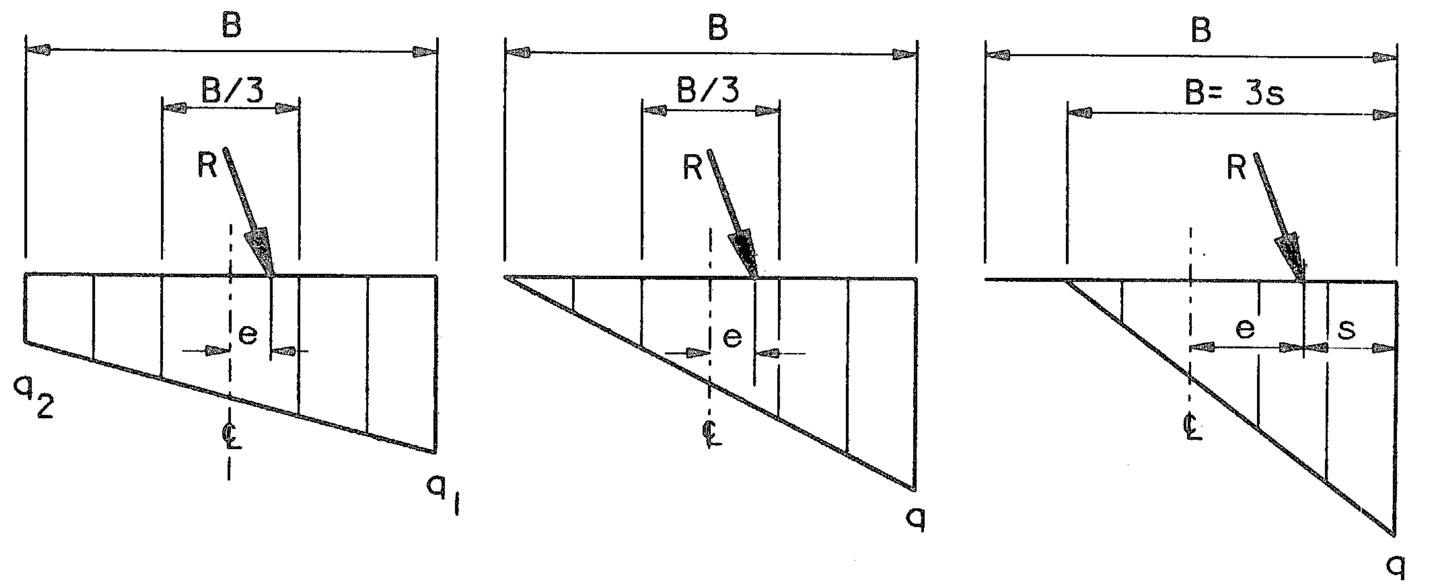
The resultant ratio is related to the percent of the base in compression as shown in Figure 4-4. The percent of the base of the structure which is in compression should be checked for compliance with the overturning stability criteria discussed in paragraph 4-9.

b. Walls with Keys.

(1) Performing an overturning stability analysis on a wall with a key requires determining the resisting forces acting along the key and along the base. Since these forces are indeterminate and cannot be determined by equilibrium methods, the following assumptions are made in order to compute the overturning stability. For a wall with a horizontal base and a key, the shearing resistance of the base is assumed to be zero and the horizontal resisting force acting on the key is that required for equilibrium, as shown in Figure 4-5. For a wall with a sloping base and a key, the horizontal force required for equilibrium is assumed to act on the base and the key, as shown in Figure 4-6. In both cases the resisting soil force down to the bottom of the toe may be computed using at-rest earth pressure if the material on the resisting side will not lose its resistance characteristics with any probable change in water content or environmental conditions and will not be eroded or excavated during the life of the wall. See examples 3 and 6 of Appendix N for stability analyses of walls with keys.

(2) Prior to performing an overturning analysis, the depth of the key and width of the base should be determined from a sliding stability analysis.

c. Sloping Backfills. For an upward-sloping backfill, an additional shear force can be taken advantage of in the overturning analysis. The calculation of this shear force is shown in Figure 4-7. The magnitude of this shear force is just large enough to cause the horizontal forces acting on the stem to be equal to the part of the horizontal wedge force that lies above the heel of the wall. This will cause the force used for the structural design of the stem to be equal to the force used in the stability analyses. This force will also cause the summation of moments about the stem-toe-heel joint to equal zero for the structural design. The derivation of this shear force is given in Appendix K. A wall with a sloping backfill is shown in example 1 of Appendix N.



100% base in compression. Resultant within central third of base.	100% base in compression. Resultant at 1/3 point.	Less than 100% base in compression. Resultant outside central third.

BASE AREA IN COMPRESSION	RESULTANT RATIO = $\frac{x_R}{\text{Base width}}$
100%	0.500 to 0.333
75%	0.25
60%	0.20
50%	0.1667
40%	0.1333

Figure 4-4. Relationship between base width in compression and resultant location

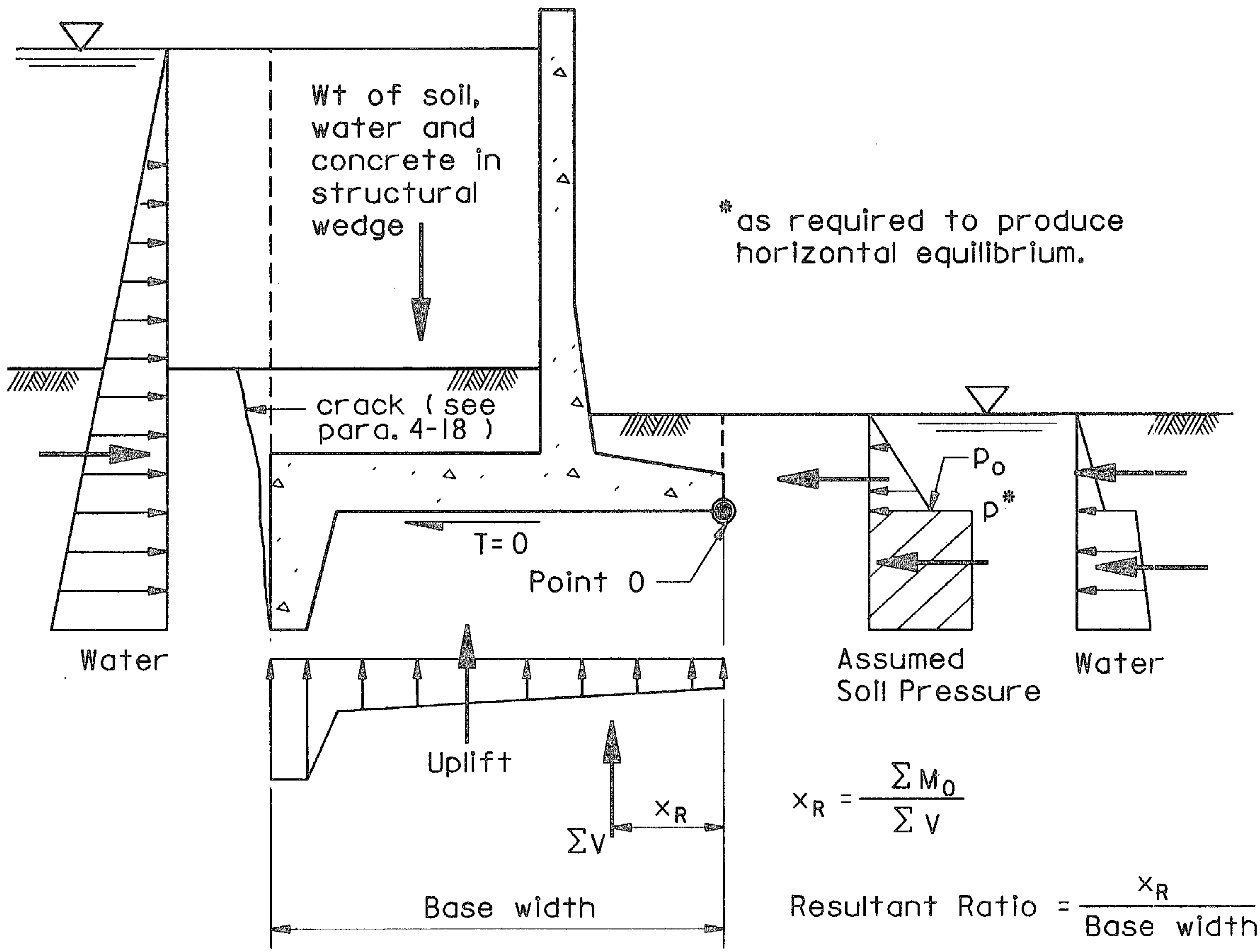


Figure 4-5. Forces for overturning analysis for wall with horizontal base and key

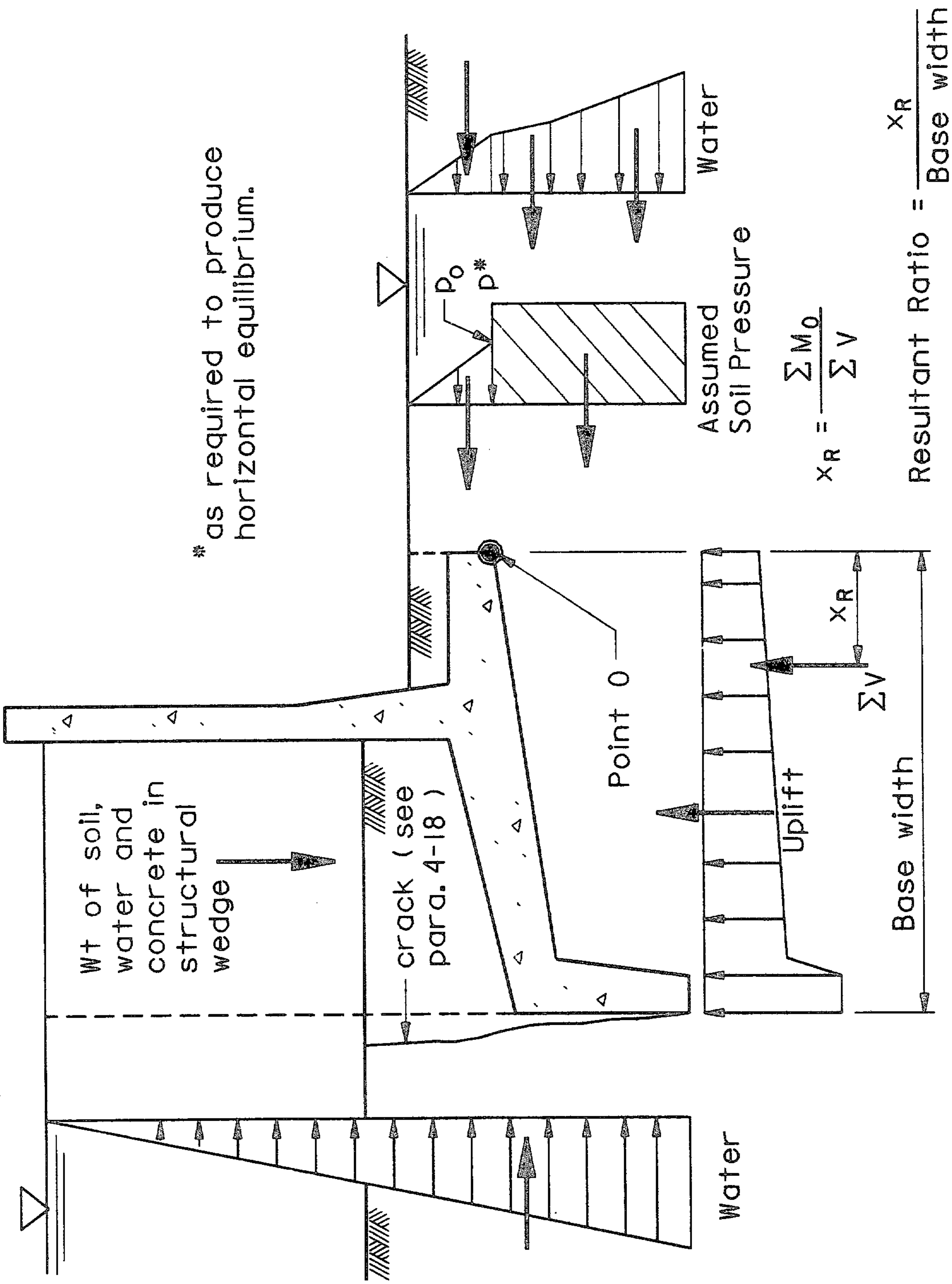
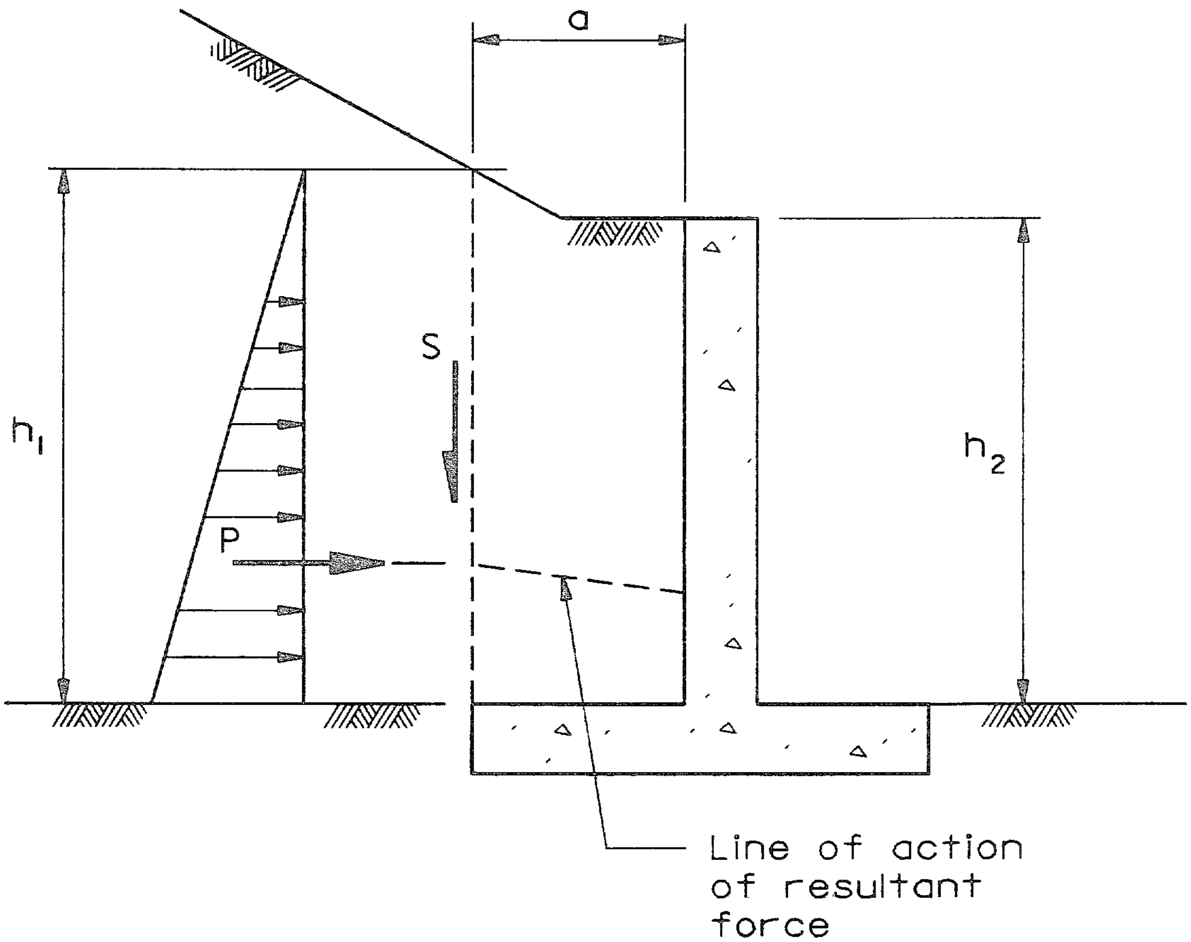


Figure 4-6. Forces for overturning analysis for wall with sloping base and key



$$S = \frac{P\,(h_1 - h_2)}{3a} = \text{Shear force when } h_1 > h_2$$

Figure 4-7. Shear force for upward sloping backfill

d. Uplift For Walls with Keys. For walls with keys, the soil may be assumed to remain in contact with the key and head loss to occur around the perimeter of the key and along the base even if the overturning analysis shows less than 100 percent of the base in compression.

4-9. Overturning Stability Criteria. The overturning stability requirements in Tables 4-1 through 4-3 are given as minimum percent base areas in compression. Figure 4-4 illustrates the relationship between the percent of the base area in compression and the resultant location.

Section IV. Structure Sliding Stability

4-10. Overview of Sliding Stability Analysis.

a. Purpose. The purpose of a sliding stability analysis is to assess the safety of a structure against a potential failure due to excessive horizontal deformations. The potential for a sliding failure may be assessed by comparing the applied shear forces to the available resisting shear forces along an assumed failure surface. A sliding failure is imminent when the ratio of the applied shear forces to the available resisting shear forces is equal to 1.

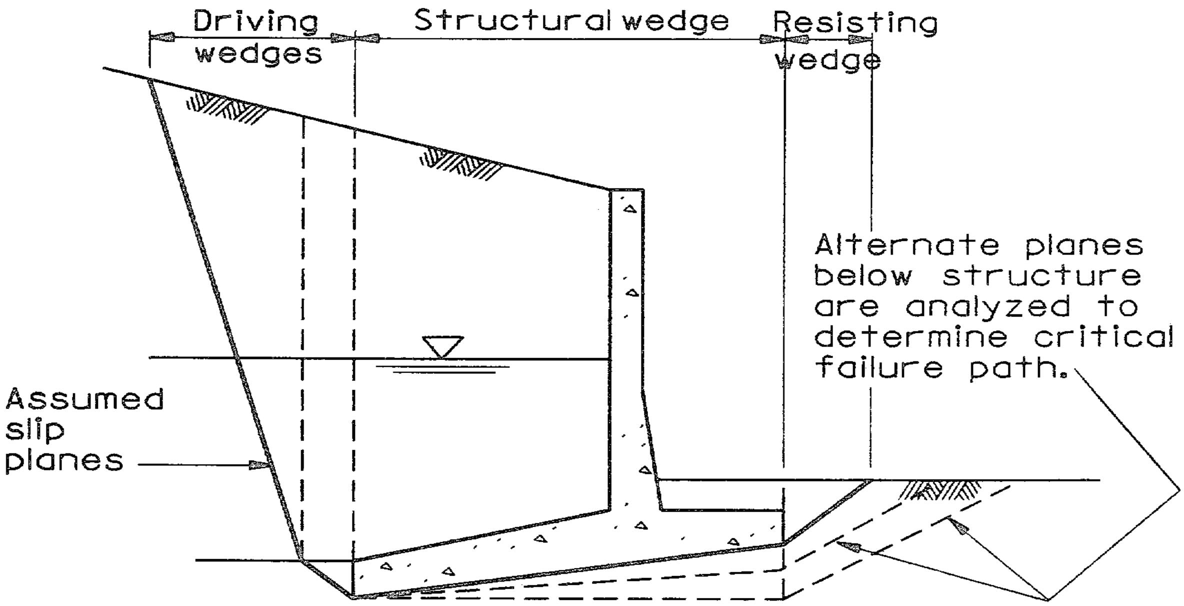
b. Analysis Model.

(1) The shape of the failure surface may be irregular depending on the homogeneity of the backfill and foundation material. The failure surface may be composed of any combination of plane and curved surfaces. However, for simplicity all failure surfaces are assumed to be planes which form the bases of wedges as shown in Figure 4-8.

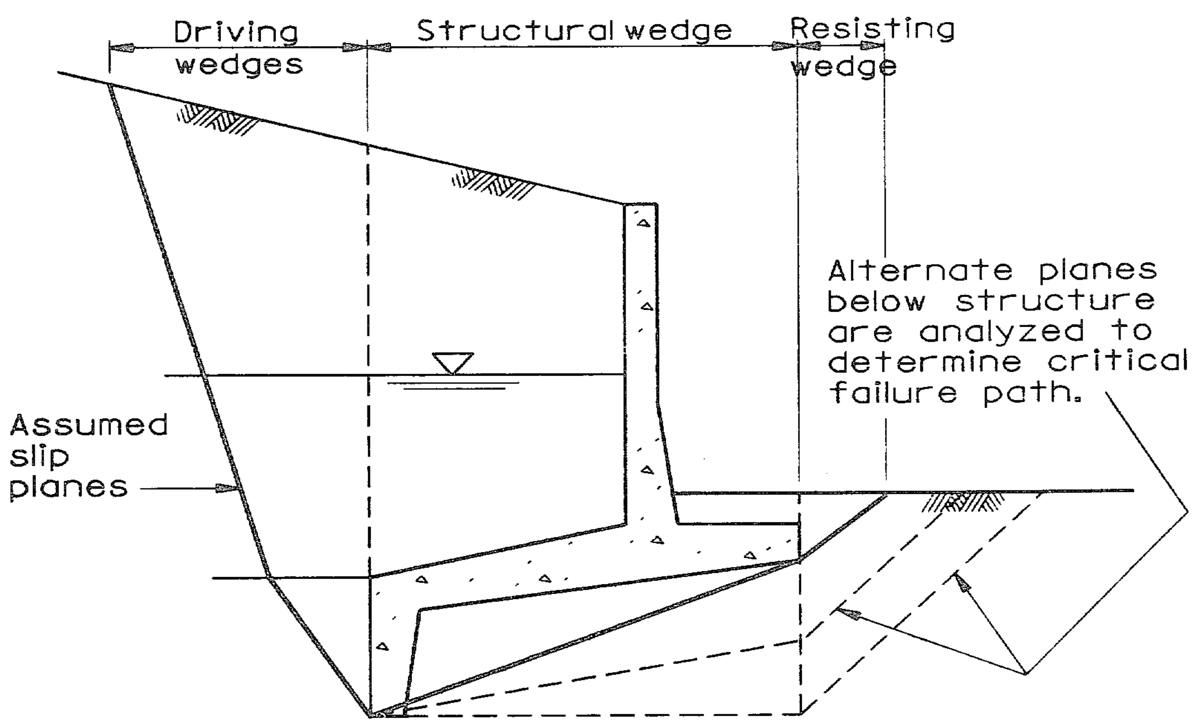
(2) Except for very simple cases, most sliding stability problems encountered in engineering practice are statically indeterminate. To reduce a problem to a statically determinate one, the problem is simplified by dividing the system into a number of rigid body wedges, arbitrarily assuming the direction of the moment equilibrium forces which act between the wedges, and neglecting any frictional forces between adjacent wedges.

(3) Figure 4-8 also illustrates how the failure surface would be divided into wedges. The base of a wedge is formed from either a section of the failure surface that lies in a single soil material or along the base of the structure. The interface between any two adjacent wedges is assumed to be a vertical plane which extends from the intersection of the corners of the two adjacent wedges upward to the top soil surface. The base of a wedge, the vertical interface on each side of the wedge, and the top soil surface between the vertical interfaces define the boundaries of an individual wedge.

(4) In the sliding analysis, the retaining or flood wall and the surrounding soil are assumed to act as a system of wedges as shown in Figure 4-8. The soil-structure system is divided into one or more driving wedges, one structural wedge, and one or more resisting wedges.



a. Failure plane for wall without a key



b. Failure plane for wall with a key

Figure 4-8. Typical soil/structure system with an assumed failure surface

(5) Depending on the geologic conditions of the foundation material, the the location of the total failure surface or parts of the failure surface may be predetermined. The inclination of some of the failure planes or the starting elevation of the failure planes adjacent to the structure may be known due to natural constraints at the site. Conditions which warrant the predetermination of parts of the failure surface include bedding planes or cracks in a rock foundation.

c. Analysis Procedure of the Soil-Structure System. An iterative procedure can be used to find the critical failure surface. For an assumed factor of safety, the inclination of the base of each wedge is varied to produce a maximum driving force for a driving wedge or a minimum resisting force for a resisting wedge. The assumed factor of safety affects the critical inclination of the base of each wedge. The factor of safety is varied until a failure surface is produced that satisfies equilibrium. The failure surface which results from this procedure will be the one with the lowest factor of safety. Several base inclinations of the structural wedge, such as those shown in Figure 4-8, should be evaluated to determine the failure surface which has the lowest factor of safety.

4-11. Sliding Factor of Safety.

a. General. Limit equilibrium analysis is used to assess the stability against sliding. A factor of safety (FS) is applied to the factors which affect the sliding stability and are known with the least degree of certainty. These factors are the material strength properties. An FS is applied to the material strength properties in a manner that places the forces acting on the structure and soil wedges into equilibrium. Since the in situ strength parameters of rock and soil are never known exactly, one role of the FS is to compensate for the uncertainty that exists in assigning single values to such important parameters. In other words, the FS compensates for the difference between what may be the real shear strength and the shear strength assumed for the analysis.

b. Definition.

(1) A state of limiting equilibrium is said to exist when the applied shear stresses are equal to the maximum shear strength along a potential failure surface. Therefore, a structure is stable against sliding along a potential failure surface when the applied shear stress is less than the available shear strength along that surface. The ratio of the shear strength to the applied shear stress along a potential failure surface is defined as the FS , as shown in Equation 4-5.

$$FS = \frac{\tau_f}{\tau} = \frac{\sigma' (\tan \phi) + c}{\tau} \qquad [4\text{-}5]$$

where

τ_f = maximum shear strength according to the Mohr-Coulomb failure criterion

τ = applied shear stress

(2) The sliding FS can also be defined as the ratio of the shear force (T_F) that would cause failure along the slip plane to the corresponding shear force (T) along the slip plane at service conditions (see Figure 4-9):

$$FS = \frac{T_F}{T} = \frac{N' \tan \phi + cL}{T} \qquad [4\text{-}6]$$

where L is the length of base in compression for a 1-foot strip of wall. For c = 0 ,

$$FS = \frac{N' \tan \phi}{N' \tan \phi_d} = \frac{\tan \phi}{\tan \phi_d} \qquad [4\text{-}7]$$

or for $\phi = 0$,

$$FS = \frac{cL}{c_d L} = \frac{c}{c_d}$$

where $\tan \phi_d$, c_d is that portion of the shear strength considered to be mobilized or developed along the slip plane(s).

4-12. Assumptions and Simplifications.

a. Slip Surface. The slip surface can be a combination of planes and curved surfaces, but for simplicity, all slip surfaces are assumed to be planes. These planes form the bases of the wedges. It should be noted that for the analysis to be realistic, the assumed slip planes have to be kinematically possible. In rock, the slip planes may be predetermined by discontinuities in the foundation. If alternate planes are possible, all must be considered to find the most critical.

b. Two-Dimensional Analysis. The sliding equilibrium method presented is a two-dimensional analysis. This method should be extended to a three-dimensional analysis if unique three-dimensional geometric features and loads critically affect the sliding stability of a specific structure.

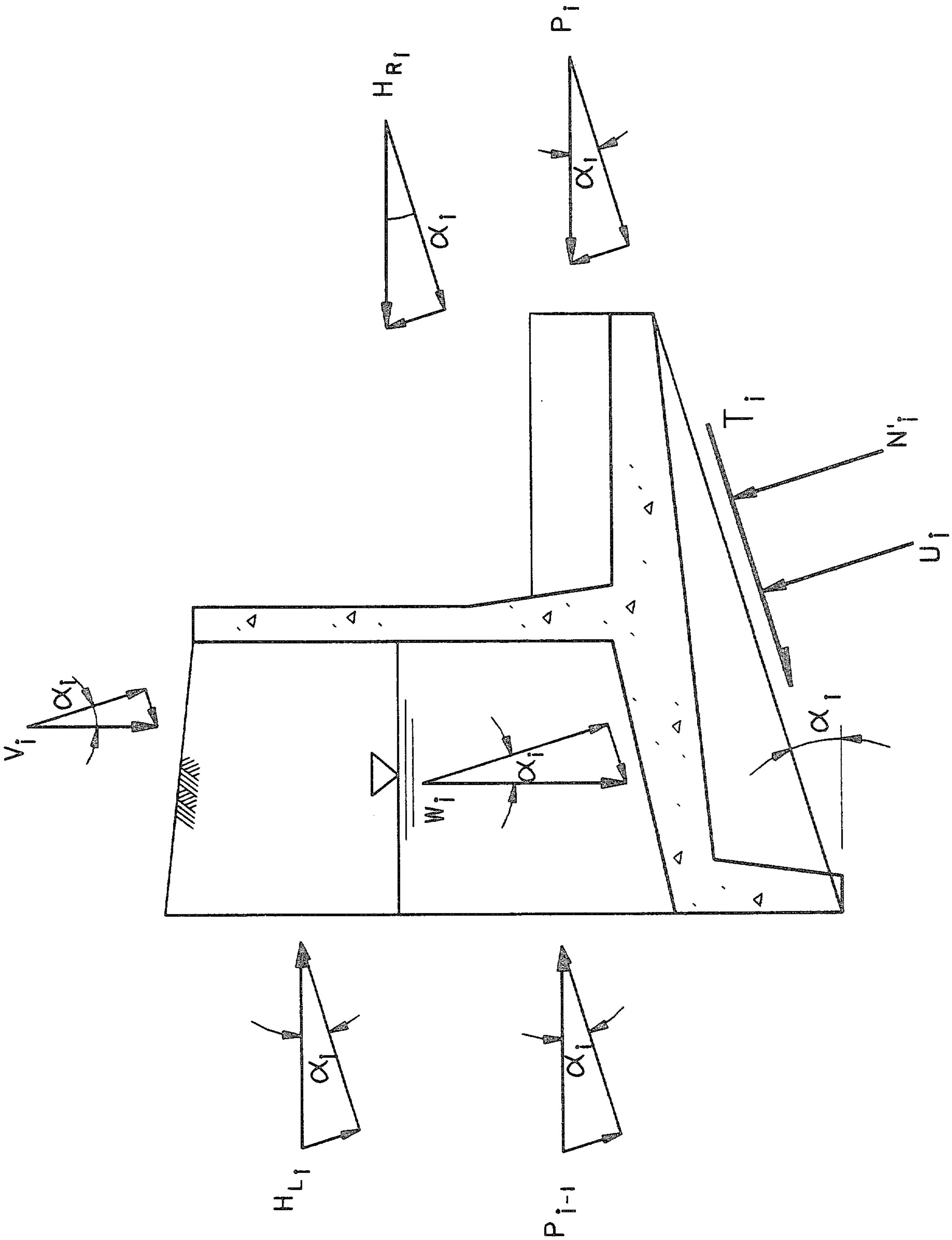


Figure 4-9. Free-body diagram of i^{th} wedge

c. Force Equilibrium Only. Only force equilibrium is satisfied. Moment equilibrium is not considered. The shearing force acting parallel to the interface of any two wedges is assumed to be negligible. Therefore, the portion of the slip surface at the bottom of each wedge is loaded only by the forces directly above or below it. There is no interaction of vertical effects between the wedges. The resulting wedge forces are assumed horizontal.

d. Displacements. Considerations regarding displacements are excluded from the limit equilibrium approach. The relative rigidity of different foundation materials supporting the structure and the concrete structure itself may influence the results of the sliding stability analysis. Such complex structure-foundation systems may require a more intensive sliding investigation than a limit equilibrium approach. The effects of strain compatibility along the assumed slip surface may be approximated in the limit equilibrium approach by selecting the shear strength parameters from in situ or laboratory tests consistent with the failure strain selected for the stiffest material.

e. Relationship Between Shearing and Normal Forces. A linear relationship is assumed between the resisting shearing force and the normal force acting on the slip plane beneath each wedge. This relationship is determined by the Mohr-Coulomb failure criterion.

f. Structural Wedge. The general wedge equation is based on the assumption that shearing forces do not act on the vertical wedge boundaries. Hence, there can only be one structural wedge since concrete structures transmit significant shearing forces across vertical internal planes. Discontinuities in the slip path beneath the structural wedge should be modeled by assuming an average slip plane along the base of the structural wedge.

g. Interface of Other Wedges with Structural Wedge. The interface between the group of driving wedges and the structural wedge is assumed to be a vertical plane located at the heel of the structural wedge and extending to the base of the structural wedge. The interface between the group of resisting wedges and the structural wedge is assumed to be a vertical plane located at the toe of the structural wedge and extending to the base of the structural wedge.

4-13. General Wedge Equation.

a. Sign Convention.

(1) The geometry and sign convention of a typical i^{th} wedge and adjacent wedges are shown in Figure 4-10. The equations for the sliding stability of a general wedge system are derived using a right-hand coordinate system. The origin of each wedge is located at the lower left corner of the wedge. The x-axis is horizontal and the y-axis is vertical.

(2) Axes which are tangent (t) and normal (n) to a failure plane are inclined at an angle (α) to the +x- and +y-axes. A negative angle is formed

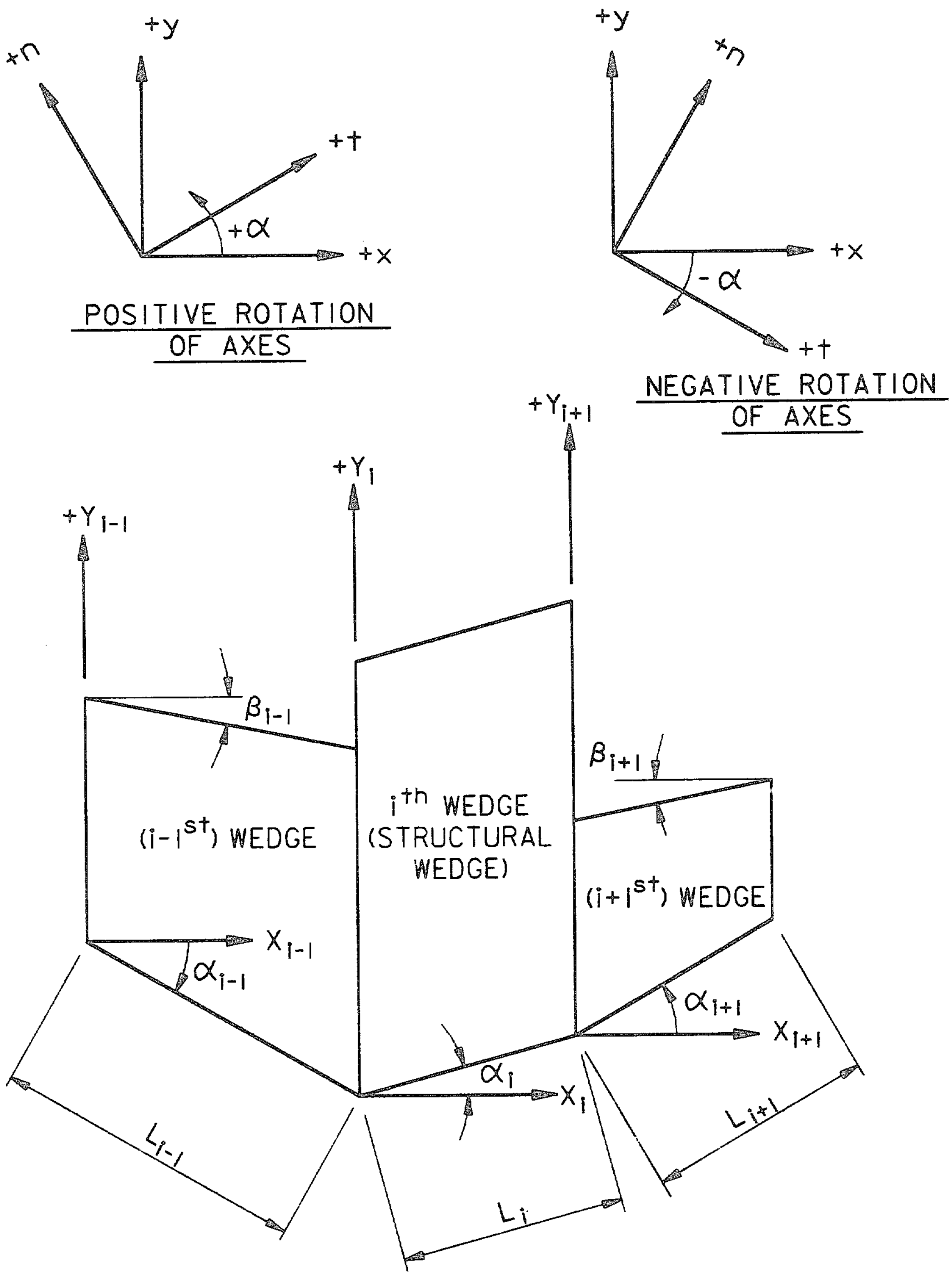


Figure 4-10. Geometry of typical i^{th} wedge and adjacent wedges

from a clockwise rotation of the axes. A positive angle is formed from a counterclockwise rotation of the axes.

b. Derivation.

(1) By writing equilibrium equations normal and parallel to the slip plane for a typical wedge as shown in Figure 4-9, solving for N'_i and T_i, and substituting the expressions for N'_i and T_i into Equation 4-6 for the factor of safety of the i^{th} wedge, the following equation results. (Refer to Appendix L for a detailed derivation.)

$$FS = \left\{ \left[(W_L + V_i) \cos \alpha_i + (H_{Li} - H_{Ri}) \sin \alpha_i + (P_{i-1} - P_i) \sin \alpha_i - U_i \right] \tan \phi_i + c_i L_i \right\} \Big/ \left[(H_{Li} - H_{Ri}) \cos \alpha_i + (P_{i-1} - P_i) \cos \alpha_i - (W_i + V_i) \sin \alpha_i \right] \qquad [4\text{-}8]$$

solving for $(P_{i-1} - P_i)$ gives the general wedge equation,

$$(P_{i-1} - P_i) = \left[(W_i + V_i)(\tan \phi_{di} \cos \alpha_i + \sin \alpha_i) - U_i \tan \phi_{di} + (H_{Li} - H_{Ri}) (\tan \phi_{di} \sin \alpha_i - \cos \alpha_i) + c_{di} L_i \right] \Big/ (\cos \alpha_i - \tan \phi_{di} \sin \alpha_i) \qquad [4\text{-}9]$$

where

i = number of wedge being analyzed

$(P_{i-1} - P_i)$ = summation of applied forces acting horizontally on the i^{th} wedge. (A negative value for this term indicates that the applied forces acting on the i^{th} wedge exceed the forces resisting sliding along the base of the wedge. A positive value for this term indicates that the applied forces acting on the i^{th} wedge are less than the forces resisting sliding along the base of the wedge.)

W_i = total weight of water, soil, rock, or concrete in the i^{th} wedge

V_i = any vertical force applied above the top of i^{th} wedge

$\tan \phi_{di} = \tan \phi_i / FS$

α_i = angle between slip plane of the i^{th} wedge and the horizontal (positive is counterclockwise)

U_i = uplift force exerted along slip plane of the i^{th} wedge

H_{Li} = any horizontal force applied above the top or below the bottom of the left side adjacent wedge

H_{Ri} = any horizontal force applied above the top or below the bottom of the right side adjacent wedge

c_{di} = c/FS

L_i = length along the slip plane of the i^{th} wedge

(2) This equation is used to compute the sum of the applied forces acting horizontally on each wedge for an assumed FS . The same FS is used for each wedge. The system of wedges is in equilibrium if the horizontal forces calculated from Equation 4-9, for all wedges, sum to zero.

4-14. Slip-Plane Angle.

a. Definition of Critical Slip-Plane Angle. The slip-plane angle α varies with the value of the FS . For a driving wedge, the critical α would be the angle that produces a maximum driving force as calculated using Equation 4-9. For a resisting wedge, the critical α would be the angle that produces a minimum resisting force as calculated using Equation 4-9. Since the determination of α is a trial-and-error procedure, for an initial trial the slip-plane angle α for a driving wedge can be approximated by:

$$\alpha = 45° + \frac{\phi_d}{2} \qquad [4\text{-}10]$$

where $\phi_d = \tan^{-1} (\tan \phi / FS)$. For a resisting wedge, the slip-plane angle can be approximated by:

$$\alpha = 45° - \frac{\phi_d}{2} \qquad [4\text{-}11]$$

b. Computation of Critical Slip Plane Angle. The above equations for the slip-plane angle are the exact solutions for wedges with a horizontal top surface with or without a uniform surcharge. Other methods to calculate the critical slip angle, for conditions other than a horizontal top surface with or without a uniform surcharge, may be found in paragraph 3-13.

4-15. Single Wedge Analysis.

a. Introduction. A quick check of the sliding stability of a structure can be obtained by performing a single wedge analysis of the structural wedge using the same loadings computed from an overturning analysis if the minimum required sliding FS is no greater than 1.5. If a minimum sliding FS greater than 1.5 is used, driving forces would be larger than the forces calculated from the overturning analysis, which uses an SMF (paragraph 3-11) of two-thirds. In this case, the single wedge equation might incorrectly indicate the structure to satisfy criteria for the larger FS (see paragraph 4-15b(5) for removing this restraint). Example calculations are shown in Appendix N.

b. Procedure for Single Wedge Analysis.

(1) Compute the sliding resistance required for equilibrium parallel to the assumed sliding plane beneath the structural wedge. Use the forces computed from the overturning analysis for the same loading case being analyzed for sliding. The sliding resistance required for equilibrium is calculated as shown in Figure 4-11.

(2) Compute the total sliding resistance available along the assumed sliding plane beneath the structural wedge using the unfactored shear strength parameters and divide the total sliding resistance by the minimum factor of safety required for the case being analyzed.

(3) If the sliding resistance needed, as computed in step (1), is equal to or less than the available sliding resistance divided by the minimum sliding factor of safety as computed in step (2), a multiple wedge analysis is not required. A multiple wedge analysis would give a sliding FS equal to or greater than the minimum required. This check on the sliding stability can be expressed by:

$$T \leq \frac{N' \tan \phi + cL}{FS} \qquad [4\text{-}12]$$

where

T = resultant of sliding resistance parallel to the assumed sliding plane required for equilibrium

N′ = resultant of forces normal to the assumed sliding plane

tan ϕ and c = unfactored shear strength parameters of the foundation material through which the sliding plane passes

L = length of sliding plane beneath the structure

FS = minimum sliding factor of safety required

If the assumed sliding plane is horizontal, T would equal the resultant of the horizontal forces and N' would equal the resultant of the vertical forces. See example 1 in Appendix N.

(4) If Equation 4-12 is not satisfied, perform a multiple wedge analysis to determine the actual sliding factor of safety (see the following paragraph).

(5) The necessity for a multiple wedge solution may be eliminated if the driving and resisting wedge forces are calculated using the minimum FS required. If Equation 4-12 is not satisfied for the FS required, a multiple wedge solution will show the same results. If Equation 4-12 is satisfied, the system has an FS equal to or greater than the minimum FS required.

4-16. Multiple Wedge Analysis.

a. Procedure.

(1) Divide the assumed sliding mass into a number of wedges, including a single structural wedge, based on the configuration and discontinuities of the backfill, wall proportions, and discontinuities of the foundation.

(2) Estimate the FS for the first trial.

(3) Compute the critical sliding angles for each wedge. For a driving wedge, the critical angle is the angle that produces a maximum driving force. For a resisting wedge, the critical angle is the angle that produces a minimum resisting force.

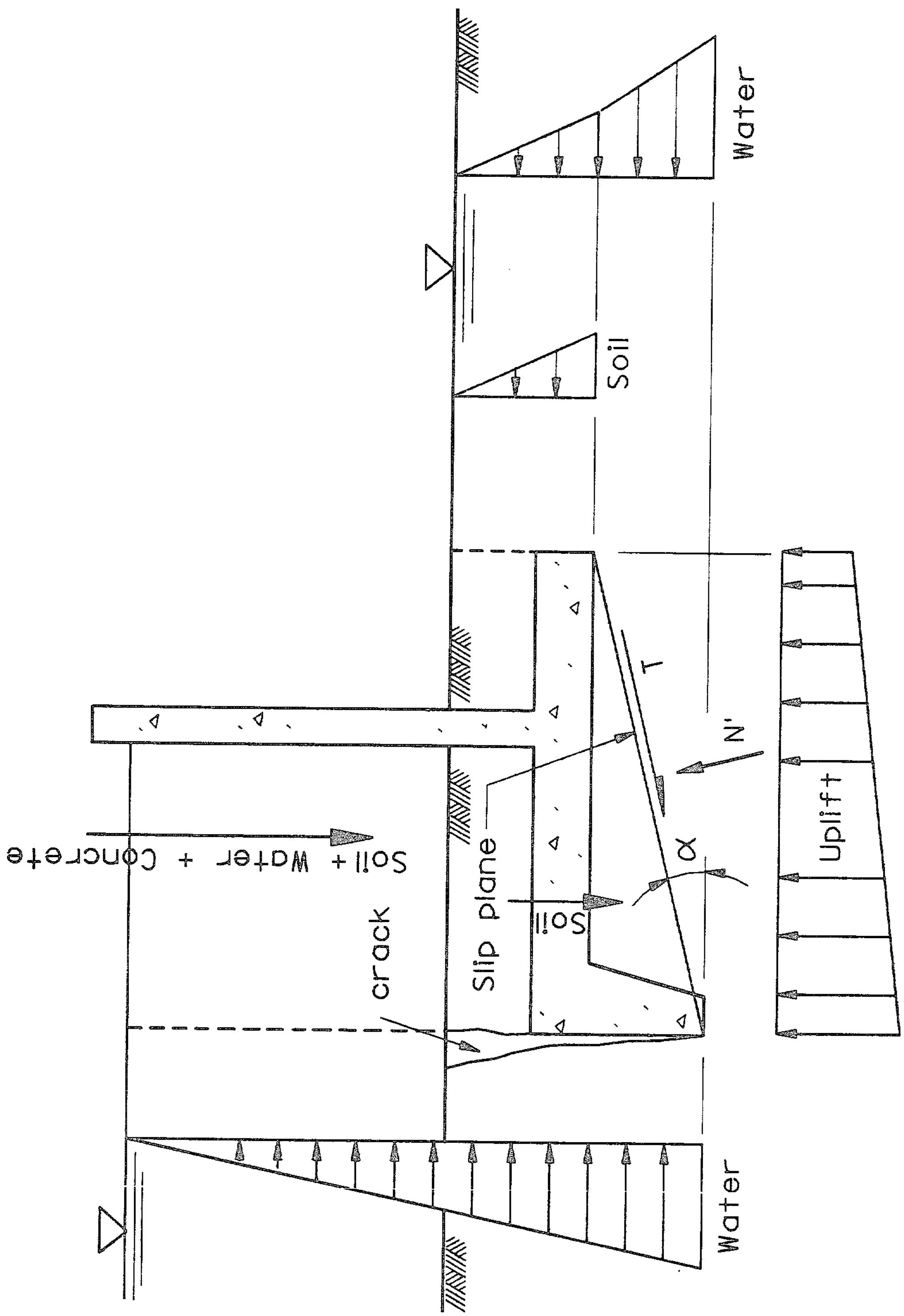
(4) Compute the uplift pressures, if any, along the slip plane. The effects of seepage should be included.

(5) Compute the weight of the wedges, including any water and surcharges.

(6) Compute the summation of the lateral forces for each wedge using the general wedge equation. In certain cases where the loadings or wedge geometries are complicated, the critical angles of the wedges may not be easily calculated. The general wedge equation may be used to iterate and find the critical angle of a wedge by varying the angle of the wedge to find a minimum resisting or maximum driving force.

(7) Sum the lateral forces for all the wedges.

(8) If the sum of the lateral forces is negative, decrease the FS and recompute the sum of the lateral forces. By decreasing the FS, a greater



$$N' = \sum V \cos\alpha + \sum H \sin\alpha \quad , \quad T = \sum H \cos\alpha - \sum V \sin\alpha$$

Figure 4-11. Single wedge analysis of sliding stability percentage of the shearing strength along the slip planes is mobilized. If the sum of the lateral forces is positive, increase the FS and recompute the sum of the lateral forces. By increasing the FS, a smaller percentage of the shearing strength is mobilized.

(9) Continue this trial-and-error process until the sum of the lateral forces is approximately zero for the FS used. This will determine the FS that causes the sliding mass to be in horizontal equilibrium.

(10) If the FS is less than the minimum required, redesign by widening or sloping the base or by providing a key.

b. Computer Program. The computer program CSLIDE (Appendix O) can assist in performing a multiple wedge sliding analysis.

4-17. Sliding Stability Criteria. The sliding stability criteria are given in terms of a minimum factor of safety for the various loading conditions as shown in Tables 4-1 through 4-3. Guidance on deep-seated sliding is given in Chapter 5.

4-18. Design Considerations.

a. Effects of Cracks in Foundation. Sliding analyses should consider the effects of cracks on the active side of the structural wedge in the foundation material due to differential settlement, shrinkage, or joints in the rock mass. The depth of cracking in cohesive foundation material with a plane ground surface can be estimated with the following equations.

$$d_c = \frac{2c_d}{\gamma' \sqrt{K_A}} = \frac{2c_d}{\gamma'} \tan\left(45° + \frac{\phi_d}{2}\right) \qquad [4\text{-}13]$$

where

$c_d = c/FS$

$\phi_d = \tan^{-1} (\tan \phi/FS)$

γ' , K_A (see Equation 3-11)

For sloping backfills see Appendix I. The value d_c in a cohesive foundation and the depth of cracking in massive strong rock foundations should be assumed to extend to the base of the structural wedge. The depth of cracking in a level clay blanket should be computed using Equation 4-13. Full hydrostatic pressure should be assumed to act at the bottom of the crack. The hydraulic gradient across the base of the structural wedge should reflect the presence of a crack at the heel of the structural wedge. Examples showing the calculation of d_c are found in Appendix N in examples 3, 4, 5, 6, and 7.

b. Passive Resistance. When passive resistance is used, special considerations must be made. Rock or soil that may be subjected to high velocity water scouring should not be used unless amply protected. Also, the compressive strength of rock layers must be sufficient to develop the wedge resistance. In some cases, wedge resistance should not be assumed without resorting to special treatment, such as rock anchors.

Section V. Bearing Capacity Analysis

4-19. General Computations. The bearing capacity is checked for the same loading conditions as determined by the overturning analysis for each case analyzed. The bearing capacity should be checked along the same plane assumed in the sliding analysis. A normal (N′) and tangent (T) force are calculated for the structural wedge along the assumed bearing plane. These forces are shown in Figure 4-11. T and N′ are used in combination to check the bearing capacity. The bearing capacity analysis discussed in Chapter 5 and in the CBEAR User's Guide (Mosher and Pace 1982) (see Appendix O) considers both the normal and tangent components of the resultant force at the base of the structure. The factor of safety against a bearing failure can be computed by dividing the normal component of the ultimate bearing capacity by the effective normal force applied to the structural wedge as shown below:

$$FS = \frac{Q}{N'} \qquad [4\text{-}14]$$

where

Q = normal component of the ultimate bearing capacity

N' = effective normal force applied to the structural wedge

The value computed from the general bearing capacity equation in Chapter 5 is the bearing capacity normal to the base of the structure. The computer program CBEAR (Appendix O) can assist in performing a bearing capacity analysis. Example calculations are shown in Appendix N.

4-20. Inadequate Bearing Capacity. If the factor of safety against bearing failure is insufficient, consideration should be given to increasing the width of the base, lowering the base of the wall, or founding the wall on piles.

4-21. Bearing Capacity Criteria. The criteria for bearing capacity are given in terms of a factor of safety as defined in paragraph 4-19 and shown in Tables 4-1 through 4-3.

Section VI. Summary of Design Procedures

4-22. Design Procedures. Figure 4-12 presents a summary of the design procedures discussed in this chapter.

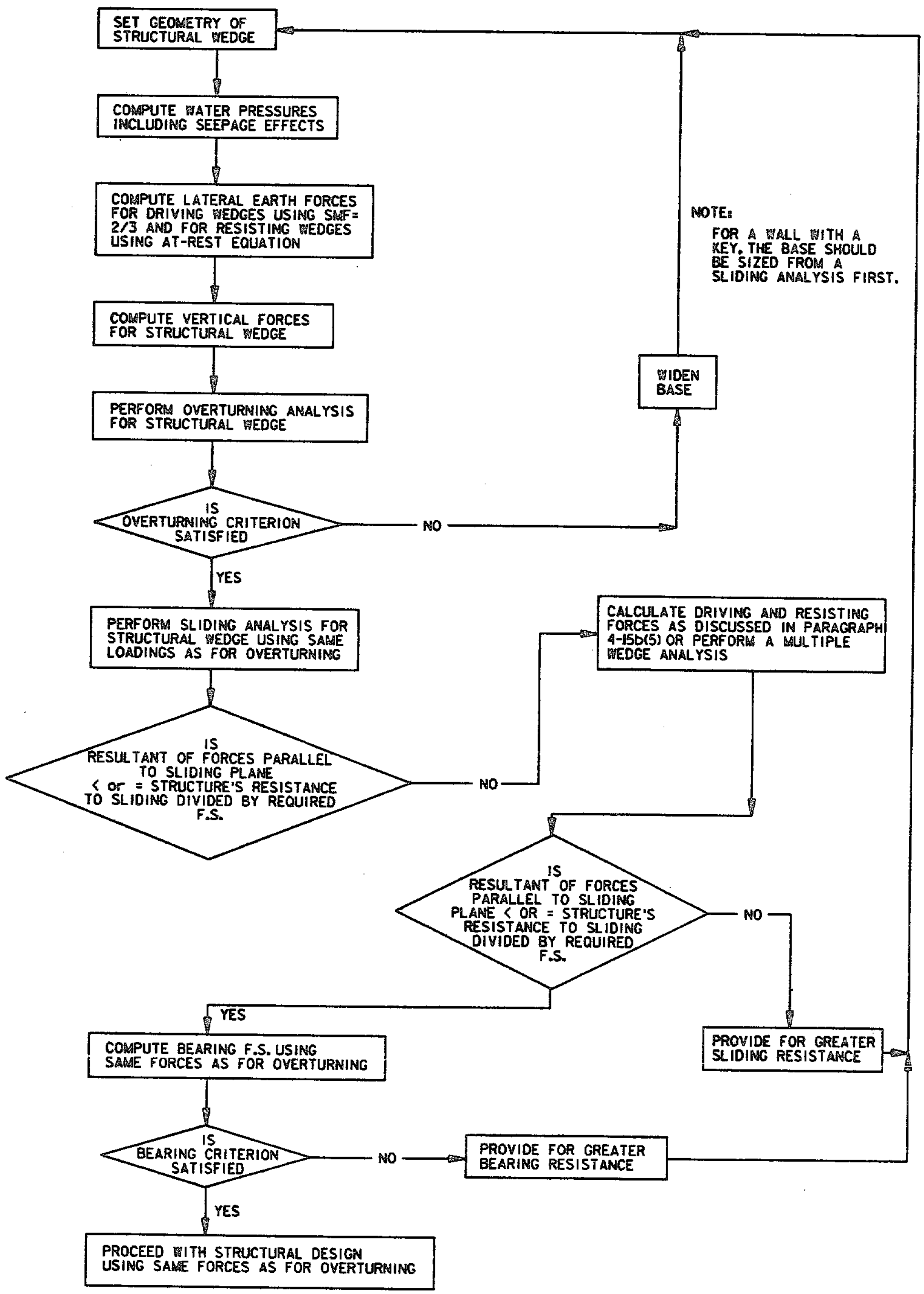


Figure 4-12. Wall design flowchart